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NUMERICAL ANALYSES OF THE PERFORMANCE OF THE DRTIJŠČICA EARTH DAM

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ABSTRACT

A regulating dam named Drtijščica was constructed in 2002. This earth dam consists of a clay core, a random fill of silty and coarse material on the downstream side, and of rock fill on the upstream side. The crest length is 265m and the dam base thickness is 150 m. The height above the lowest foundation level is around 20 metres. The subsoil consists of a layer of loose, highly compressible clayey-silty material, three to five meters thick, overlying bedrock made of tectonically altered clayey shists and siltstones. During construction of the dam, settlements of the subsoil were measured at eight locations and after the construction a larger monitoring system was established.

The paper deals with the analysis of the results of measurements taken during and after the construction of the Drtijščica earth dam. Back-analyses of the performance of the dam during construction were made using FEM calculations. The numerical analyses were performed in plane strain conditions using the Mohr-Coulomb material model as well as two soil models which take into account the stress-dependency of soil stiffness. In the paper the results of calculated settlements for drained and undrained loading and measured settlements are presented.

INTRODUCTION

The construction of a new motorway in a very narrow valley of the waterstream Radomlja has diminished the flow safety of that current. The decision was made to construct a regulating dam on a waterstream Drtijščica, which is a left tributary of the Radomlja in the lower part of the valley. A 670m long water tunnel of 4.0m diameter was excavated between the valley of the Radomlja and the Drtijščica. The high waters of Radomlja will be directed through the tunnel, retained at the Drtijščica regulating dam and gradually let again into the waterstream Radomlja.

A characteristic cross-section of the dam is presented in Fig. 1. This earth dam consists of a clay core (the material designated with No. 2 in Fig. 1). On the downstream side it consists of a random fill of silty and coarse material (material No. 4), and with rock fill (material No. 3 and No. 1) on the upstream side and at the bottom on the downstream side. The crest length is 265 m and the dam base thickness is 150 m. The height of the dam above the lowest foundation level is around 20 metres, and the absolute elevation a.s.l. of the top of the dam immediately after construction was 357.9m. The normal operating water level is to be 13.1m below the crest level but during rainy periods the headwater level may reach a height of no less than 3.3m beneath the dam crest. The subsoil consists of a layer of loose, highly compressible clayey-silty-sandy material, three to five meters thick (named 'Alluvium' in Fig. 1), overlying bedrock made of tectonically altered clayey shists and siltstones.

The Drtijščica dam set of buildings consists of the earth embankment, a reinforced concrete surface intake structure, a bottom gate structure, a bottom outlet, a spillway (stone in lean concrete), a sealing curtain and a conduit pipe system for watering a habitat for frogs. The spillway is expected to be in function only exceptionally, when the inlet gets clogged-up with debris during heavy rainfall. Two small bridges and a number of culverts had to be constructed for a local road that leads around the future lake.

PRELIMINARY INVESTIGATIONS

As preliminary works for the design of Drtijščica dam, ten geological boreholes were drilled, with a total length of 95m, of which 37m were drilled in clayey schists. Five intact samples were taken from the very compressible layer of clayey silts with lenses of sandy silt (named 'Alluvium' in the text). Three oedometer tests and three residual shear tests were conducted on the samples of subsoil.

The characteristics of the clayey core, i.e. highly plastic silt (MH), from a nearby cut on the motorway route were determined on six soil samples. The plasticity limits, dry and natural unit weights were established and CBR tests were made on three samples.

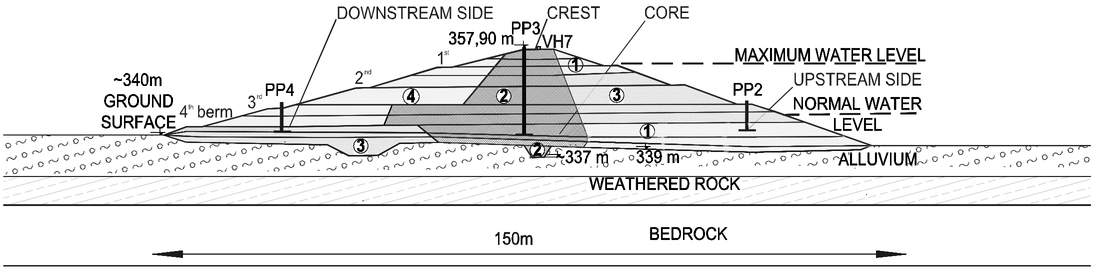


Fig. 1. Characteristic cross-section of the Drtijščica dam

The material from these samples was compacted according to the Proctor optimum density. On these cylinders three oedometer tests were conducted as well as one triaxial shear strength test and four direct shear tests.

An earthquake of 8th degree on the European macroseismic scale can be expected in this area with an expected acceleration of the ground of 0,26g for the return period of 500 years.

CONSTRUCTION

The construction works of the dam began in February 2001, the crest height was reached in November 2001 and the works were finished in February 2002. The time-elevation diagram of the filling works at the location of the settlement plates (see Table 2) is shown in Fig. 2. Actually, the construction of the right side of the dam began later on, due to the construction of the bottom outlet. The layout of the dam as well as the locations of the settlement plates and some of the benchmarks are shown in Fig. 3.

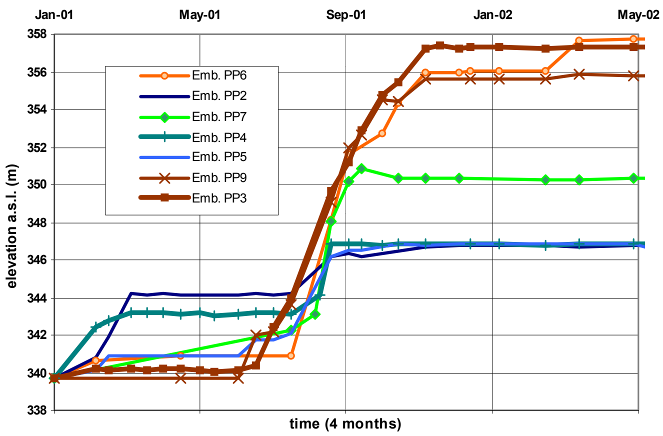


Fig 2. – Time–elevation diagram of the dam construction

The quality of the construction of the earth dam as well as of all other structures was regularly checked according to the Quality Control plan. On the upstream side, crushed rock from the nearby quarry was compacted using vibrating rollers. Two categories of material were brought. One was named CR I (pure crushed rock) and the other CR II (crushed rock with some clay). Highly plastic silt (designated '2-Core' in the text) was brought directly from a cut on the motorway route. For the compaction of the clayey core tamping foot rollers were used. For the bottom layers of the downstream side, material from the quarry was used (CR I). Later on tailing material from the quarry and from the cut, was used (the material designated '4-downstream side' in the text). Because of the mixing of two different materials the in situ density and other soil characteristic varied locally a lot on the downstream side of the dam.

The earth material was compacted in layers 40-50cm thick. The material samples were taken regularly to establish the optimum moisture content (standard and modified Proctor test). The in-situ density of the soil was measured by the nuclear method. On the layers of the core material 755 tests were performed, on the downstream side 79 tests and on the upstream side 280 tests were performed. In Table 1 are presented the results of the quality control findings on compaction efficiency.

Table 1. Results of measurements of compaction efficiency

	Labor a-tory	Labora-tory	In situ	In situ	In situ compa-
	$\rho_{d,max}$	$w_{opt.}$	ρ_d	w	ction
	Mg/m^3	%	Mg/m^3	%	%
2-Core	**1.405**	**31.1**	**1.375**	**36,1**	**97.2**
Min	1.312	29.0	1.298	27.8	92.3
Max	1.441	37.8	1.479	43.9	103.7
4-Downstream			**1.906**	**19.6**	**97.4**
Min			1.320	5.7	93.0
Max			2.395	39.4	103.4
3-Upstream	**2.368**	**5.1**	**2.248**	**8.0**	**97.4**
Min	2.259	3.3	2.122	3.0	92.3
Max	2.415	6.8	2.396	14.6	102.4

DESIGN PREDICTIONS

According to the design calculations, the maximum expected settlement of the foundation ground (designated 'Alluvium') was 33 to 38cm at the location of the dam crest.

For the assessment of the consolidation of the foundation ground the coefficient of permeability was evaluated. The results of permeability tests varied a lot. A coefficient of permeability $k=5.6*10^{-10}$ m/s was estimated from the oedometer consolidation curve, and $k= 5*10^{-11}$ m/s from the falling head permeability apparatus. So the shortest possible consolidation time for the subsoil was six months and the longest possible (but not likely) was five years.

The safety of the dam was determined for three load cases: A - normal water head in the retaining basin, B - maximum waterhead, C - load case immediately after drawdown of the maximum water head with a high water level in the core (all cases without dynamic loads). The calculated factor of safety for the case A was 2.0, for B 2.1 and for C 2.0.

Table 2. Data on the settlement plates

plate No.	Plate (m)	Embank-ment	Differen-ce (m)	Settleme nt (cm)	
PP4	341.4	347.1	5.7	25	down-stream
PP7	342.1	350.9	8.8	23	down-stream
PP3	340.2	357.7	17.5	52	core
PP9	341.7	356.3	14.6	46	core
PP6	340.8	350.9	10.1	44	core
PP2	340.8	346.8	6.0	25	core
PP8	340.7	346.0	5.3	17	up-stream
PP5	340.0	346.6	6.6	21	up-stream

MEASUREMENTS

The validity of design estimations of the magnitude of the subsoil settlements was checked during and after construction of the dam. Settlements were measured using nine settlement plates, which were installed one to two metres above the original ground level. One settlement plate was demolished during construction and several were damaged. Table 2 shows data on the absolute elevation a.s.l of each installed plate, the final elevation of the embankment at the location of the plate, the thickness of the embankment (above the plate) and the measured final settlement of each settlement plate. The last column gives a description of the location of the plate (two of them are located beneath the 3rd berm on the downstream side, two beneath the 3rd berm on the water side and four in the core beneath the crest). Settlement of the foundation ground on the downstream side was measured during construction also in a 54m long profile using a hydrostatic horizontal inclinometer.

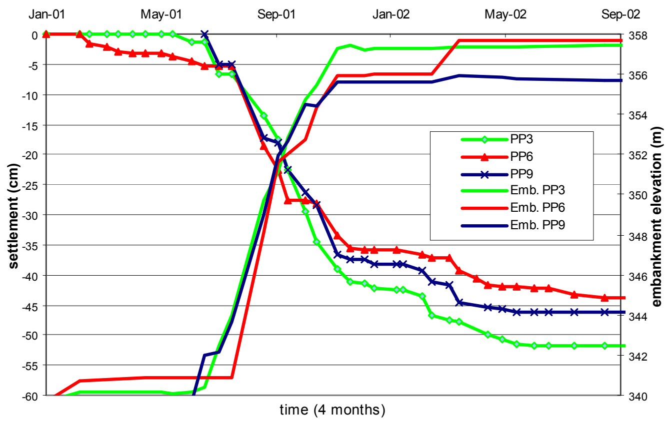


Fig. 4. Time/settlement diagram for the plates in the core

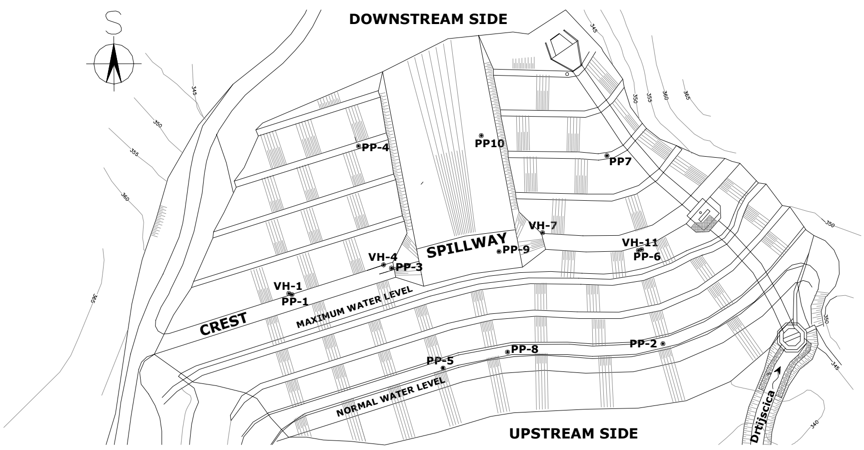


Fig. 3. – The layout of the dam and locations of the settlement plates

Measurements of vertical displacements started with the installation of the settlement plates and were taken approximately twice a month. The elevations of the crest and time/settlement curves for plates PP3, PP6 and PP9 in the core are shown in Fig. 4. The consolidation of the subsoil took approximately six months.

Since the dam height exceeds 15 metres, it is defined as a large dam according to the World Register of Dams. With regard to the national regulations, the structural behaviour of large dams has to be regularly monitored. Three months after the crest height had been reached, the planned monitoring system was established and in February 2002 the datum measurements were taken. Monitoring includes measurements of the piezometric head in nine piezometers, of pore pressures at two locations in the clay core and of the horizontal displacements beneath the surface in four vertical inclinometers. The datum measurement of twelve benchmarks (X,Y,Z) was not taken until December 2002.

Fig. 5. – The stage of dam construction reached in September 2001

BACK CALCULATIONS

Since the technology of construction did not follow the original design, and materials other than those planned were incorporated on the ‘4-downstream side’, the measured settlements were larger than those obtained by the numerical model used in the design. It was therefore necessary to estimate the future behaviour of the dam in order to assess the expected values of the measured parameters by the established monitoring system.

We analysed the results of measurements taken during and after the construction of the earth dam. We performed FEM numerical analysis using three different material models. In the back analyses the stiffness parameters of the in situ subsoil, earth and rock fill of the dam were fitted in order to get better matching between the calculated and the measured values of settlements. We modelled the dam as it was actually constructed – the characteristics of the incorporated materials, the technology and the timescale of the dam construction, as well as the findings of Quality Control.

Material models and parameters

For the assessment of dam safety and settlements in the design phase, calculations were made using the Plaxis program (version 7.2) in plane strain conditions using the Mohr-Coulomb material model. Table 3 shows the material parameters used in the design phase, which were chosen on the basis of laboratory tests and experience.

Table 3. Material characteristics in the design FEM calculations

Layer	γ kN/m^3	c' kPa	φ' ^{o}	k m/day	E_{ref} MPa
Alluvium	19.0	5	24	$9\cdot10^{-4}$	4
Weathered shists	25.0	0	36	$9\cdot10^{-4}$	30
Shists	25.0	0	40	$9\cdot10^{-3}$	200
1- Downstream side CR I	21.0	0	36	100	60
4 – Downstream side - silty clay	18.0	10	18	$9\cdot10^{-4}$	10
2 - Core - silty clay	18.0	10	18	$9\cdot10^{-6}$	10

Back analyses were performed using the same FEM code. Back calculated material properties are given in Tables 5 and 6. The numerical computations were performed using the Mohr-Coulomb material model as well as a so-called 'Soft soil model' and 'Hardening soil model', which take into account the stress-dependency of soil stiffness. The material models and parameters are described in detail in the PLAXIS manual.
We chose a stress-dependent soil model only for the highly compressible soil layers ('2-core' and 'Alluvium'). For the less compressible soils (1- upstream side, shists, 4-downstream side) we used the Mohr-Coulomb model in all analyses.
First we assumed that all materials were drained. We tried to match the calculated and the measured magnitude of the final settlement (see Table 7). Reliable strength parameters of the incorporated core materials were not available. Therefore we used various combinations of stiffness and shear parameters for the materials 'Alluvium' and '2-Core'. Alternatively we assumed that the materials were undrained (which is a more realistic assumption) and predicted the final settlement and consolidation of the dam itself. So we had seven different calculations cases, as shown in Table 4. The designations of the load cases used later in the text are shown in the last two columns.

Table 4. Material models used in the back analyses

Soil model	Soil layers	Tables	Drained	Un-drained
Mohr-Coulomb - design	all	3	1D	
Mohr-Coulomb –back analyses	all	5	2D	2U
Soft soil	'Alluvium' and '2 core'	5, 6	3D	3U
Hardening soil	'Alluvium' and '2 core'	5, 6	4D	4U

Input data

The differences between the design and back analyses input parameters are as follows:
The phreatic line is lower in the back analyses. In the design calculations the normal water head during operation of the dam was assumed but during most of the construction time the Drtijščica waters were kept inside the watercourse and the major part of the valley was waterless. One year and a half after the construction the situation is still the same, so we assumed a lower phreatic line. This change causes larger settlements.

Based on the quality control of compaction, we calculated the actual natural weight γ of the incorporated materials (see Tables 1 and 5). In the design it was assumed that the core and the downstream side would be constructed of plastic silt. Since there was a shortage of suitable material, the downstream side was constructed of tailing material.
On the basis of the soil investigations, experience and measurements taken, the material characteristics for the Mohr-Coulomb material model as shown in Table 5 were assumed. The characteristics that are different from those used in the design calculation are printed in bold. Poisson's coefficient ν=0.3 was adopted for all soil types. An angle of dilatancy was assumed ψ=6° for the crushed stone (CR I), and ψ=0° for the other soil types.

Table 5. Material characteristics – the Mohr-Coulomb soil model

Layer	γ kN/m^3	c' kPa	φ' °	k m/day	E_{ref} MPa
Alluvium (clays, silts)	19.0	5	24	$9\cdot10^{-4}$	**3**
Weathered shists	25.0	0	36	$9\cdot10^{-4}$	30
Shists	25.0	0	40	$9\cdot10^{-4}$	**100**
1–upstream side (crushed stone I)	**22.7**	0	36	100	60
3–upstream side	**22.7**	**0**	**30**	**100**	**40**
4–downstream side	**21.0**	**10**	**20**	**$9\cdot10^{-3}$**	**20**
2–core (plastic silt)	**18.7**	**10**	**22**	$9\cdot10^{-6}$	**10**

The embankment load caused immediate settlement of the compressible subsoil, as well as settlement due to later consolidation. Since the settlement plates were installed up to two meters above the original ground level, part of the settlement of the subsoil and the embankment developed before the initial (datum) measurement of the heights of the settlement plates. In Tables 7 and 8 the total calculated settlements are presented, but of course they are not strictly comparable with the results of the above-mentioned measurements.
For the clayey core layers up to two meters above the original ground level, we assumed that the compaction was less effective than higher up in the dam. This was a realistic assumption since the measured density and % compaction was lower in these layers since the clayey trench in the middle of the core (see Fig. 1) was narrow and the original ground was soft. We assumed a previous overconsolidation pressure of POP=50 kPa for the lower layers and POP=200 kPa for the largest part of the core. That was a very rough assumption, since we had not investigated the core material after it was compacted into the dam.

We did not measure the compression of the dam itself, and the characteristics of silty clay were assumed on the basis of tests conducted on reconstituted samples.
The stiffness properties of ‘Alluvium’ and ‘2-core’ for the ‘Soft soil’ and ‘Hardening soil’ material models are shown in Table 6.

Table 6. Material characteristics – Soft soil and Hardening soil

Soil model, load case, layer	λ*	κ*	ν_{ur}	*POP* (kPa)
Soft soil 3D, 3U Alluvium	0.040	0.008	0.15	0
Soft soil 3D, 3U 2 core	0.0146	0.0041	0.15	200 (50)
	E_{50} (MPa)	E_u (MPa)	E_{oed} (MPa)	*POP* (kPa)
Hardening soil 4D, 4U Alluvium	2.3	7.8	2.9	0
Hardening soil 4D, 4U 2 core	9.0	27.0	11.0	200 (50)

A Poisson’s coefficient of ν_{ur}=0.15 and a power m=0.5 were used for the ‘Soft soil’ and ‘Hardening soil’ model, respectively.
The time effects due to cyclic loading and unloading during filling and discharging of the retaining basin as well as viscous effects were not considered in the numerical modelling. Only consolidation is taken into account in the analyses.

Calculated displacements

Firstly drained analyses were performed to obtain the final settlements. The results are presented in Table 7 for three different material models and four characteristic locations together with the measured values of settlement at these locations (see Fig. 1 and Fig. 3). The three characteristic locations correspond to the locations of the settlement plates (PP3 in the core at the elevation of 342m, PP2 under the 3rd berm on the upstream side and PP4 under the 3rd berm on the downstream side). The fourth characteristic location for the comparison of settlements is the crest of the dam at an elevation of 357.5m. The measurement of the settlement of the crest started four months after the construction was completed, so the actual value of the total crest settlement is not known.
In the second set of analyses we assumed that the soil layers with low stiffness and permeability ‘2-core’, ‘Alluvium’ and ‘4-downstream side’ were undrained.

Table 7. Drained conditions – calculated settlements (in cm)

material model	1D MC Design	2D Mohr-Coulomb	3D Soft soil	4D Hard. Soil	measured values
core	27	50	40	41	~45cm
crest	50	74	69	64	>70cm
upstream	11	22	24	23	~20
downstream side	10	24	26	25	~25

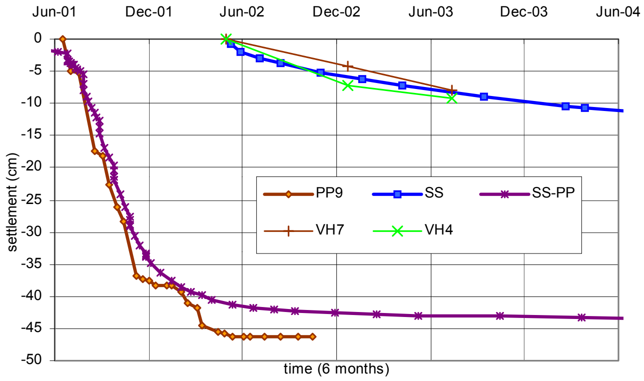
From Fig. 2 we can see that it took approximately nine months for the crest to reach its final height. In the analyses filling of the dam was performed in nine construction stages or layers from 0.8m to 2.0m thick. Each undrained construction stage was followed by a consolidation stage of appropriate duration. The total consolidation time, considered in staged construction calculation was 270 days, which corresponds to the actual construction period.
Table 8 presents the calculated settlements for the three material models, immediately after the construction of the dam (t=0) and final settlements. The settlements are shown for four characteristic locations (the same as in Table 7).

Table 8. Undrained conditions – calculated settlements (in cm)

material model	2U MC	2U MC	3U Soft soil	3U Soft soil	4U Hard Soil	4U Hard Soil
	t=0	Final	t=0	Final	t=0	Final
core	37	52	34	42	34	45
crest	66	93	55	94	66	98
upstream side	22	23	24	25	24	24
downstream side	22	23	24	25	25	25

All the models with undrained loading gave higher final settlements than were obtained in the simple drained analysis. This is attributed to the distortion settlements that occur during undrained loading. The consolidation of the subsoil calculated from the FEM analyses followed the observed behaviour well. The calculated settlement of the crest after construction of the dam ranges from 27cm to 39cm. In Fig. 7 the final displacements of the Drtijščica dam as calculated using the ‘Hardening soil’ model are shown.
Fig. 6 shows the measured settlement of the subsoil at the location of the plate PP9 (marked “PP9”) and the settlements at the same location calculated using the ‘Soft soil’ material model (marked “SS-PP”).
Additionally, Fig. 6 presents the settlement of the crest for the first two years after construction as calculated using the ‘Soft soil’ model (marked “SS”) and the measured values

(benchmarks on the crest – marked “VH4”, “VH7”). The measured consolidation of the dam core matches the results of the calculations quite well.



6. Time settlement diagram of the subsoil and crest (calculated and measured)

Prediction of dam behaviour

The expected behaviour of an earth dam and its safety is desired information for the manager and the owner of the structure. In case of large dams the manager is usually obliged to perform monitoring of the structure. For effectual monitoring one needs to know the expected values of the monitored parameters. Only with analyses of the structure as actually constructed (incorporated materials, loads during construction, time plan) is it possible to evaluate these parameters. It is necessary to specify the expected values of the parameters to be monitored, which comply with the expected loads. When the magnitude of these values is known one is able to define the optimal scope of the monitoring and to define the necessary accuracy of the measurements to be taken. In the case of the Drtijščica dam the monitoring system was planned in the design stage so the characteristic values were not defined at that stage.

We calculated the deformations (horizontal and vertical displacements) and safety of the dam for the same load cases as in the design calculations: A - normal water head (a rise of the phreatic line compared to the previous cases), B - maximum water head, C – after the quick drawdown of the maximum water head. The calculated deformations are due to the rise of the water level only. In the calculations we assumed, that the dam is already consolidated. The displacements due to dynamic loads were not calculated, since the planned monitoring system comprises only occasional measurements.

Table 9 presents the calculated maximum displacement of the dam crest for the three load cases (change in the water level) and three soil models. The Mohr-Coulomb (MC) soil model takes into account the same stiffness modulus for loading and unloading, so the calculated displacements are overestimated.

Table 9. Calculated displacements (in cm) and the factor of safety

Load case Soil model	U2 MC	U3 Soft soil	U4 Hard. Soil
A max displ.	**3.5**	**2.8**	**2.9**
A max horiz.	2.3	0.9	1.2
A max vert.	3.1	2.8	2.8
A safety	*2.2*	*2.2*	*2.2*
B max displ.	**11.1**	**6.5**	**7.6**
B max horiz.	9.2	4.3	5.5
B max vert.	6.9	5.7	5.9
B safety	*2.3*	*2.3*	*2.4*
C max displ.	**4.0**	**3.2**	**3.4**
C max horiz.	2.1	1.5	1.5
C max vert.	3.7	3.2	3.3
C safety	*1.9*	*2.1*	*2.2*

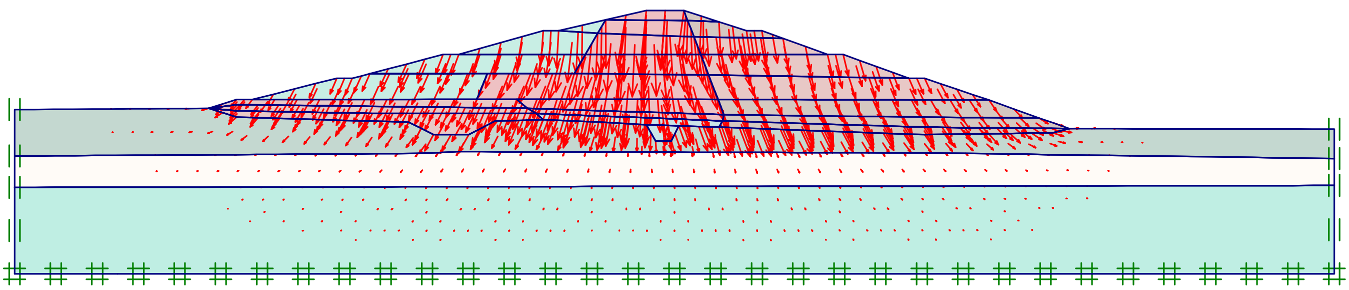


Fig. 7. Final displacement of the Drtijščica dam (‘Hardening soil’ model)

CONCLUSIONS

Back analyses of the behaviour of earth structures may be time consuming and without reliable input data they are often meaningless. In most cases, however, back analyses prove to be a powerful tool for accurate predictions of the behaviour of earth structures based on previous monitoring results and other reliable input data. The behaviour of the executed dam will change in time due to consolidation, cyclic loading and unloading during filling and discharging of the retaining basin and viscous effects.

Experts in geotechnical engineering should perform careful planning of geotechnical investigations and monitoring system during and after construction as well as back analyses. Still it is very important that the person involved is familiar with the actual conditions at the site and with all minor technical changes that could influence the behaviour of the structure.

For the determination of the final settlement simple equations and the Mohr-Coulomb soil model could give reasonably accurate results.

For the estimation of the behaviour of the structure during construction and operating time, and especially for the determination of displacements, FEM analyses together with more sophisticated material models should be used. In case of cohesive and compressible materials, a model that takes into account the stress-dependency of material stiffness, consolidation during load placement and viscous effects should be used. For a number of reasons (soil testing, limitations of material models, heterogeneity of earth structures, …) a numerical model is always only an approximation of a real structure and can never reproduce exact in-situ behaviour.

The values of the parameters to be monitored during the operating time of the structure should be determined on the basis of back analysis after the structure is constructed and results of measurements are available.

Back analysis of the Drtijščica earth dam showed that the calculated settlements of the subsoil match the magnitude and the development of the measured settlements well. The measured consolidation of the dam core matches the results of the calculations for undrained loading quite well. It is also possible to predict the expected deformations of the dam crest during its operating time. To better model the consolidation and secondary compression of the dam core, the model should be calibrated on the basis of the results of future monitoring.

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